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# JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS

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## ENGINEERING AND CONSTRUCTION FEATURES OF THE TENNESSEE RIVER VALLEY SYSTEM

By HARRY A. HAGEMAN \*

(Presented at a meeting of the Boston Society of Civil Engineers held on September 22, 1937)

THIS is an unexpected privilege and pleasure for me to speak to the members of this Society in part on a well-known subject, namely, the Tennessee Valley Authority. Having lived in the vicinity of Boston for many years it is only natural that I should come back to New England to spend my vacation, and to renew acquaintance with my friends of former years, especially those of the engineering fraternity.

Shortly after my arrival here recently I received a telephone call from Mr. Carl A. Bock, assistant chief engineer of the Authority, asking me to represent Dr. Arthur E. Morgan, chairman of the Authority Board, who was unable to address you tonight, and to speak on certain phases of the Authority's work in the Tennessee River Valley. I was very glad to accept this assignment, and propose to discuss briefly some general engineering and construction features of the hydroelectric plants that have been planned, some of which are completed or are under construction.

The Tennessee Valley Authority was authorized by an act of Congress in 1933. This act is administered by a board of three, Messrs. Arthur E. Morgan, chairman; Harcourt A. Morgan, assistant chairman, and David E. Lilienthal, director. The primary purpose of the act is to

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\* Chief hydraulic engineer, Tennessee Valley Authority.

unify the development of the Tennessee River system by constructing dams and reservoirs to aid and stimulate navigation on the main river and its tributaries and control the destructive floods originating in the watershed. The act also directs the Authority to provide facilities at any dam, as may be consistent with the regulation of the stream flow, to generate hydroelectric power, — such power to be transmitted and marketed as provided by the act, to aid in liquidating the costs or the maintenance of the projects.

The tributary streams which form the Tennessee River have their source in the mountains of eastern Tennessee, western Virginia, North Carolina and northern Georgia. The drainage area from the source to its mouth at Paducah, Kentucky, where it empties into the Ohio River, is approximately 40,600 square miles. This drainage area, especially in the mountains, is productive of a high rainfall with a quick run-off, thereby creating large floods, with the consequent damage to the cities and towns in the valley and the land in the watershed.

The river from Knoxville, Tennessee, to Paducah, Kentucky, falls about 500 feet in 650 miles. In this stretch of river will be located the principal structures, such as dams to form the reservoirs for controlling the floods, locks to aid navigation in reaching the higher levels, and power plants to produce hydroelectric energy. In the higher elevations dams will be constructed on the most important tributaries to create flood storage reservoirs, the water from these reservoirs being released at certain seasons of the year, through the dams and water wheels of the power plants, to augment the low summer flow to aid navigation and to generate large blocks of power to supply any deficiency in the transmission system.

The plan contemplates the construction of eight main river developments and three tributary river developments. Commencing farthest downstream and coming up the river, the developments proposed or under construction and those existing are as follows:

1. Gilbertsville (proposed).
2. Pickwick Landing (under construction).
3. Wilson (generally known as Muscle Shoals, constructed during the World War period).
4. Joe Wheeler (completed and dedicated this month).
5. Guntersville (under construction).
6. Hales Bar (existing and owned by the Tennessee Electric Power Company).
7. Chickamauga (under construction).



8. Watts Bar (site now being explored).

9. Coulter Shoals (proposed).

On the tributary rivers the following developments are either existing, under construction or proposed:

1. Norris (completed).

2. Hiwassee (under construction).

3. Fontana (proposed).

The main river plants are practically all of the low head type, the average heads being from 40 to 92 feet. Of the tributary river plants, the Norris development, near the former junction of the Clinch and Powell rivers, has a head varying from 129 to 194 feet. The Hiwassee Dam will be nearly 300 feet high, and the proposed Fontana Dam will develop a maximum head of about 382 feet.

This system of hydroelectric developments will create large reservoirs for flood control storage, having a capacity of six million acre-feet on the main river and two million acre-feet on the tributaries. Of this storage capacity the proposed Gilbertsville reservoir will contribute three and one-half million acre-feet. The operation of these reservoirs under a plan of unified control will substantially aid in checking the Mississippi River floods and provide at least two feet additional free-board to its levees.

The system will also provide for the future installation of power-generating equipment at the several dams totaling about 2,000,000 kilowatts. Of this capacity about 1,650,000 kilowatts would be in the main river plants, and about 350,000 kilowatts would be supplied by the plants on the tributaries.

The geological formation of the Tennessee River and its tributaries has been investigated with great care and reports have been issued on over one hundred dam sites. The age and type of rocks differ materially. The river bed in eastern Tennessee and northern Alabama consists largely of dolomite, limestone, sandstone and shale. In some cases these materials are fairly soluble, and sinks and caves are not unusual; in other cases these rocks are satisfactory for dam foundations. From the southern to the northern boundary of Tennessee the right bank of the river is bordered by limestone of good quality, while the left bank consists of sands and clays, and does not show rock except at considerable depths. Through Kentucky to its junction with the Ohio River the present Tennessee is flowing over ancient river channels of sand, gravel and clay material from 100 to 300 feet thick. However, at the Gilbertsville site, in this stretch, the rock is at a reasonable depth below the river bed.

At most of the main river developments the foundation problems are extremely difficult and costly to solve for water tightness.

The structures for the main river developments are similar in character. In general they all contain or will have a navigation lock, an intake and power house section, a gate-controlled spillway, and in most cases an earth dam connecting the concrete structures with the river shores.

The distance across the structures comprising a project, from shore to shore, varies from a few thousand feet up to 9,000 feet. The spillways will have ample capacity to discharge safely the largest flood flows which may occur. The maximum recorded flood of 450,000 cubic feet per second occurred in 1897 at the Gilbertsville site. The spillway for this project will be designed to discharge safely 960,000 cubic feet per second.

These spillways will be controlled by steel gates. The gates, for most developments, will be mounted on rolled steel flanged wheels equipped with heavy-duty roller bearings and run on tracks in the gate slots. The gates will be provided with water seals of moulded rubber for water tightness. There will be two gates in each spillway opening, each 20 feet high by 40 feet long, placed one above the other and operated by an electrically driven traveling gantry crane of 80 tons capacity. Two cranes will be provided for each spillway. These cranes run on tracks over the entire length of the spillway and power house intake sections. There are some exceptions to the type of spillway gates just described; these are as follows: the spillway gates for the Wheeler development are of the radial type, mechanically operated; those for the Norris development are of the drum type, hydraulically operated; and those on the Wilson Dam are of the sliding type, hydraulically operated.

The water flowing over the spillways will fall into a water cushion of variable depth, depending on the river stage, on to the heavily reinforced concrete apron connected with the spillway. This apron, in most cases, will be about 85 feet long and 900 feet wide, depending upon conditions at the site, and will have a suitable weir at its downstream end to insure that the hydraulic jump will take place on the apron rather than on the river bed below it.

The Authority has constructed a well-equipped hydraulic laboratory at Norris, Tennessee, where extensive model tests have been made for all hydraulic structures. The information obtained from these tests has been invaluable in working out an economical design of the structure in question. The intake structure will have in general a three-channel waterway to supply the scroll case of each water wheel unit. Each intake will have the customary trash screen, the bars having a 9-inch spacing



to keep out the heavy débris floating in the reservoir. Means will be provided to remove large floating material and not pass it on to the next downstream plant. The trash handling problem will have careful attention.

All intakes will have the usual gates to provide a means for inspection or maintenance of the waterways supplying the turbines. The trash racks and intake gates will be operated by the gantry cranes referred to above.

The water wheel units for the low head plants will be of the vertical-shaft, high-speed-propeller Kaplan type. Each water wheel unit will have a runner about 25 feet in diameter and be directly connected to a 60-cycle, 3-phase, 13,800-volt generator. This voltage will be stepped up in most cases to 154 kilowatts for long distance transmission.

Much thought has been given to the selection of the speed of the units. The specific speed chosen for the water wheels has been determined from extensive model tests and experience records of existing units to minimize cavitation. Considerable investigation has been made regarding the lengths, sizes and shapes of the water passages to and from the turbines in the interest of high efficiency, low maintenance and smooth operation. The generating units will each be from about 25,000 to 36,000 kilowatt capacity, depending on the plant. On account of the relatively low head and large capacity of the units, the principal component parts will be of large physical dimensions. For example, the Pickwick Landing wheels, now nearly completed, have a speed ring 37 feet in outside diameter and weighing about 240,000 pounds. The main shaft is 36 inches in diameter. The outside diameter of the generator housing is over 40 feet. These large capacity units require steel castings and forgings of large dimensions, which only comparatively few companies in the country can furnish. These companies at present are extremely busy; and it is not only difficult to obtain proposals from them, but the delivery time is long. The so-called high head plants will have vertical-shaft, Francis-type water wheel units of large capacity. For example, the Norris plant has two 50,000 kilowatt units in operation since last year, and it is proposed to install two 60,000 kilowatt units in the Hiwassee plant. These two plants will probably operate on about a 20 per cent annual load factor basis and can supply large blocks of power to the entire system, if necessary, during the low flow season.

The power plants which the Authority has built so far are of reinforced concrete. It is now proposed to build the power house superstructures for the Chickamauga and Guntersville plants of brick, above the high tail water level; and studies are now in progress on this basis.

It is believed that a more pleasing effect can be obtained with the superstructure of brick construction, and there is a material saving over concrete.

Much is being done on standardization of design, especially on the Chickamauga and Guntersville developments. The head difference in these developments varies by only a few feet, making it possible to use the same design for the spillway and intake sections and the power house, modified as to reinforcement to suit the small difference in head of the two plants. The deck girders spanning the spillway openings and supporting the traveling gantry cranes for operating the gates are alike in both cases. The spillway gates and gantry cranes will be alike, also the power house cranes. Trash screens, intake gates and draft tube gates will have the same design for each type of equipment. The design for the power houses will generally be the same. A cross section of the power houses shows that the distance from the intake gates to the center of the water wheel units and the length of draft tubes are the same. The reinforcing design has been standardized to a large extent by making loading and stress diagrams for both developments, from which a typical design is made showing the spacing, sizes and arrangement of the reinforcing steel. There are many other items, including oil, water and air systems, railings, lighting fixtures, etc., which are being standardized for both jobs. It is impossible to make both developments exactly alike since there are minor differences which must be taken into account, but enough standardization has been done to conclude that large money savings will be made in design and the purchase of equipment.

The area covered by the structures has been extensively explored by core borings, and the site selected largely from this information. Generally each development on the main river is built in three stages. A sheet-pile, cellular-type cofferdam is used in the river area, the earth embankment being constructed on the dry land prior to making the closure in the river section.

An interesting core-boring machine is being used, making it possible to take out rock cores from 36 inches in diameter, and larger if necessary. While most of the information regarding the foundation is obtained from small rock cores, say 3 to 4 inches in diameter, the 36-inch diameter core holes enable one to descend into the hole and observe the character of the rock at close range below the foundation surface. The area within the cofferdam having been unwatered, the foundation is carefully prepared and inspected prior to placing the reinforcement and pouring the concrete.

The preparation of the foundation for the earth embankments is done with equal care. The foundations for all structures must be practi-



cally water-tight by sealing them, and thereby preventing water from flowing through the structure and endangering the stability.

In general, two lines of holes are drilled close to the upstream face of the concrete structures, the holes being 10 feet on centers and inclined about 20 degrees from the vertical. The second line of holes is drilled 5 feet downstream from the first line, the hole spacing being the same but staggered with the first line and inclined in the opposite direction. The hole depth depends upon conditions, but usually varies from 20 to 30 feet.

After the holes are cleaned and tested they are grouted at about 15 pounds pressure per square inch, care being taken to watch the pressure and check the foundation for movement. Another line of holes much deeper is drilled about 10 feet downstream from the second line, these holes being tested and grouted at much higher pressure after the concrete dam has been poured. This grout curtain is the first line of defense against serious leakage under the structure, and the indications are that it will be entirely satisfactory. For the earth embankments, the excavation for the concrete core wall foundation is taken down to solid rock. Holes for grout are then drilled at about 4-foot centers, and after being cleaned and tested the grouting is done under variable pressure, as required. If the rock is not water-tight, the 4-foot spacing is cut in two and the grouting done as before. Usually this is effective, although in many instances the spacing has been reduced to 1-foot centers before the rock could be made tight. Where caverns have been encountered it has been necessary to resort, in some cases, to mining operations; that is, sinking a shaft and driving a smaller tunnel to the affected area. These caverns usually contain sand and gravel. After all loose material has been removed, the hole is cleaned and sealed with concrete.

Each job is equipped with a concrete testing laboratory, in charge of a technician who determines the concrete mixture and does the necessary testing. There is also a geologist on the job to study the rock structure of the foundation and advise the engineering and construction forces accordingly.

The construction plant has been carefully designed to meet the conditions. All equipment is thoroughly modern and of large capacity. Much thought has been given to the design of the plant and the selection of the equipment for manufacturing, transporting and placing concrete. This is equally true as regards the handling and placing the fill in the earth embankments. The results obtained indicate that both the concrete and the earth fill have been built into the structures in a satis-

factory and workmanlike manner. Much of the equipment, such as concrete mixing plants, storage bins, tractors, bulldozers, pile drivers, cable-ways, etc., is being used from job to job until worn out or otherwise disposed of.

Four of the developments above mentioned, including the privately owned Hales Bar plant, have been completed and are in operation. The Wilson plant has been operating for many years. The Norris development was completed and put into operation early in 1936. The Joe Wheeler development was finished this year and dedicated this month. The Hales Bar plant has been operating for about thirty years. There are under construction at present the following major developments: Pickwick Landing, Guntersville, Chickamauga and Hiwassee. These will be completed in the order given. The first named will go into operation next summer; the other three at about one-year intervals thereafter.

When the program outlined above is completed there will be available about 5,800,000,000 kilowatt hours of primary energy and about 1,000,000,000 kilowatt hours of secondary energy 80 per cent of the time in an average year.

The 1936 report to Congress, as submitted by the Board of Directors, states in regard to expenditures substantially as follows:

The suggested program of dam construction now under way and recommended by the Tennessee Valley Authority for the unified development of the Tennessee River system, which would be most efficient from an engineering standpoint, would require an appropriation of about \$35,000,000 a year for between seven and eight years, in addition to investment which may be necessary for generating electric power at other than the Norris, Wheeler and Pickwick Landing Dams.

In closing let me say that the unified development of the Tennessee River system as proposed will, when completed, accomplish at least these three important things:

1. It will greatly assist navigation throughout the year by providing an adequate channel with an ample water supply from Knoxville to Paducah, thereby promoting commerce by water transportation between the ports on the Tennessee River and those on the Ohio and Mississippi rivers.
2. It will contribute in a large way in assisting to control the floods on the lower Ohio and Mississippi rivers.
3. Lastly, it will provide a large amount of electrical energy at a low cost to a great population within the valley and several hundred miles from it.



ENGINEERING GEOLOGY OF THE PASSAMA-  
QUODDY PROJECT

BY IRVING B. CROSBY, MEMBER \*

(Presented at a meeting of the Designers Section of the Boston Society of Civil Engineers held on  
October 14, 1936)

*The Passamaquoddy Tidal Power Project has been before engineers in one form or another for many years, and although the project was abandoned in the summer of 1936 it seems desirable to place on record the results of the extensive geological and foundation investigation made for this project. Interest in this record is increased by the fact that the dam sites have certain similarities and certain strong contrasts with dam sites elsewhere in New England. — EDITOR*

## INTRODUCTION

THE peculiar features of the many dam sites of the Passamaquoddy tidal power project are due to the fact that these dam sites, unlike most dam sites, have been under the sea in post-glacial times. This project, near Eastport, Maine, has been before engineers in various forms for over ten years, but it was not until 1935 that any work beyond the most preliminary investigating was done on the project.

This paper will be entirely confined to the technical aspects of this problem and largely to the relation of geology to design and construction. Before taking up the geology, some of the engineering features of the project will be considered briefly.†

The original scheme of Dexter P. Cooper comprised two tidal basins: Passamaquoddy Bay, largely in Canada, and Cobscook Bay, in the United States. The power house would have been between these and continuous power would have been possible. (Fig. 1.) This international scheme was temporarily abandoned some years ago on account of opposition of the Canadian government. Several schemes entirely

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† For a more detailed description of the engineering, plan of operation, and purpose of the project, see "Construction of Rock-Filled Dams in Flowing Water on the Passamaquoddy Tidal Power Project," by Hugh J. Casey, Second Congress on Large Dams, Communication No. 3, Washington, 1936.

in the United States were then proposed and the project was adopted by the United States government in May, 1935, and assigned to the Corps of Engineers, United States Army, for construction.\* The writer was appointed consulting geologist of the project.

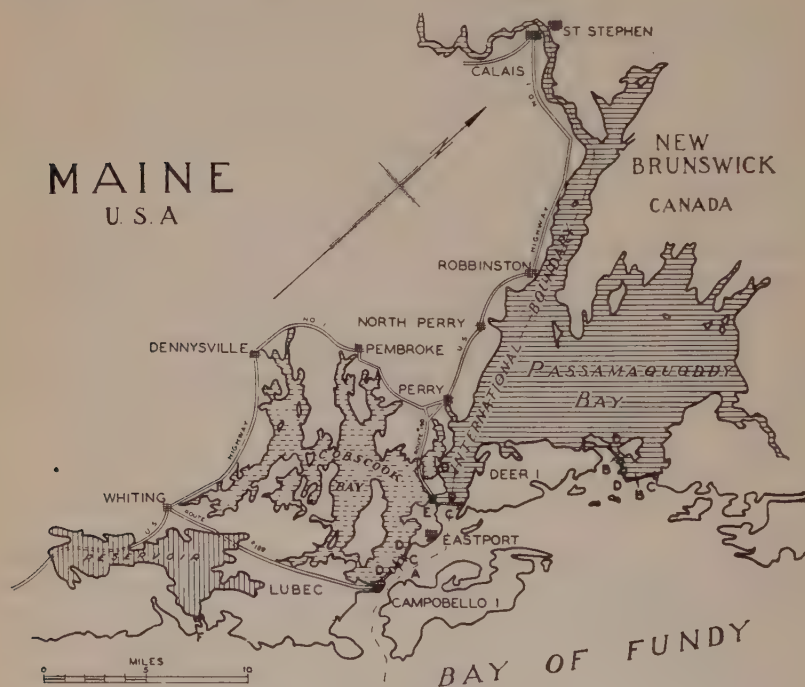


FIG. 1. — LAYOUT MAP OF PASSAMAQUODDY TIDAL POWER PROJECT, INTERNATIONAL PLAN

- |                            |                           |
|----------------------------|---------------------------|
| A. Filling gate structure  | D. Dam                    |
| B. Emptying gate structure | E. Power house            |
| C. Navigation lock         | F. Pumped storage station |

(From Communication No. 3, Second Congress on Large Dams, Washington, D. C., 1936. Paper by Hugh S. Casey, Captain, Corps of Engineers, United States)

The plan, as now modified from Cooper's original scheme, is to enclose Cobscook Bay from the Bay of Fundy and from Passamaquoddy Bay, by a series of dams, gate structures, navigation lock, and power house, and provide for intermittent power generation between the high-tide level of Cobscook Bay and the fluctuating level of the Bay

\* Lieut.-Col. Philip B. Fleming was District Engineer and Capt. Hugh J. Casey was Chief of the Engineering Division.



of Fundy. The tides at Eastport have a mean range of 18.1 feet, ranging for extreme perigean spring tides up to almost 27 feet, and for apogean neap tides to less than 13 feet. The cycle of operation would be as follows: At and near high tide the filling gates would be opened, permitting the pool to fill to near high-tide level. As the tide begins to fall below the pool level, the gates would be closed. When the tide falls to  $5\frac{1}{2}$  feet below the pool level the turbines would be started and power generated during the ensuing falling and rising tide until a head of  $5\frac{1}{2}$  feet was again reached. The power station would then be shut down due to insufficient head. As the rising tide reaches the level of the interior pool, the gates would again be opened to restore the draw-down in the pool to near high-tide level.

Such operation would provide only intermittent power, about  $7\frac{1}{4}$  hours out of each 12-hour and 25-minute tidal cycle. Cooper's plan for the priming of this power was by provision of a pumped storage reservoir (known as the Haycock Harbor Reservoir), about sixteen miles distant, where energy would be stored during peak output and drawn on during shut-down periods. This and another reservoir site were extensively investigated, but due to a 45 per cent loss of power and a large duplication of pumping and generating facilities the pump storage reservoir was very costly. The Army Engineers, therefore, recommended provision for priming by (a) interchange of power with existing hydro-electric facilities, (b) construction of supplemental hydro-electric facilities, or (c) stand-by steam or Diesel electric plant.

The initial development contemplated 10 units developing 15,000 kilowatts each at 23-foot head. The power house could be expanded to provide for 20 such units, or a total of 300,000 kilowatts. The power house is located near a neck of land between Cobscook and Passamaquoddy bays. It therefore fits into a potential international two-basin development providing for a continuous power development.

Under the latter arrangement, Passamaquoddy Bay would also be cut off from the Bay of Fundy by a series of dams, navigation lock, and gate structure, but Passamaquoddy Bay would be maintained as a low-level basin, its gates being opened at and near low tide. Power could thus be generated continuously through the interconnecting power house, head being maintained by restoration of draw-down in the high-level Cobscook Bay pool at high tide, and by draining off the rise in tail-water from the low-level Passamaquoddy Bay pool at low tides.

When work on the project was stopped a year ago extensive investigations and surveys had been made for the tidal dams and power house, gate structures and lock, and for two alternate pump storage

reservoir sites. Geological investigations had been made of sixty dam sites. No actual construction work had been done on either of the two pump storage reservoir sites when this feature of the project was withdrawn from consideration. Three small tidal dams had been built and the site of the lock and gate structures had been cleared when construction ceased.

### GENERAL GEOLOGY

The geology of the Passamaquoddy area is extremely complicated. Volcanic lava flows and tuffs and sedimentary beds were folded and cut by faults and intruded by igneous rocks, producing a very intricate pattern. This was subjected to stream erosion for a long period and then glaciated, following which the area was submerged beneath the sea to about 200 feet above present sea level and marine deposits were laid down. Each chapter of this long, complex history had its influence upon the engineering project, but the effects of glaciation and marine submergence are of the greatest importance.

Deposition of the rocks of this area began in the Silurian age, several hundred million years ago. The sedimentary rocks were deposited in a sea as muds, sands and gravels, from which shale, slate, sandstone, quartzite conglomerate and limestone were found, and the lavas were poured out from volcanoes. The igneous rocks include diabase (trap-rock) flows, tuffs and intrusions, and rhyolite (felsite) flows and tuffs and some andesite. Subsequent to their formation the rocks were folded and faulted in an intricate manner, with the result that the geologic map is extremely complex. Faults are very numerous, and evidence of shearing is often found where there is no distinct fault. As a result much of the diabase and rhyolite is very badly fractured.

After the formation of these rocks the region was raised above the sea and stream erosion worked upon this complex mass for a long time. The resulting topography was partly controlled by the rock structure. The streams took advantage of zones of faulting and of belts of soft shales, leaving the harder rocks as ridges. Where the belts of the various formations were curved the resulting valleys were curved. This curving of the topography is very pronounced in Cobscook Bay.

The great ice sheet modified this topography, bevelling off some of the hills and widening and possibly deepening some of the valleys, but its greatest effect was by partially blanketing the hard rocks with glacial deposits, filling many of the old valleys and greatly changing the appearance of the topography. The glacial deposits consist of boulder clay, clay, silt, sand and gravel.

During the glacial period the weight of the ice depressed the surface of the earth under it several hundred feet, also the level of the seas of the entire earth were lowered some 300 feet, due to the great amount of water locked up in the ice sheets. As the ice melted the sea rose and the surface of the earth in the glaciated regions was tilted, the uplift being greatest to the north where the weight of the ice and the resulting depression had been greatest. As a result of these movements we find elevated beaches about 200 feet above the present level of the sea in the vicinity of Eastport, Maine, but the highest elevated beaches in Massachusetts are a hundred feet or more lower.\* While this area was submerged marine deposits of clay and sand were laid down upon the glacial deposits and upon the hard rocks partially concealing them.

The submergence beneath the sea at the close of the glacial period resulted in an intricate shore line. The sea invaded valleys and interior low areas producing Cobscook and Passamaquoddy bays and the innumerable small bays and inlets which indent the shore. Due to complete disarrangement of the preglacial drainage by the glacial deposits, the present irregular drainage has little relation to the old drainage. Streams have often cut down upon buried rock ridges while near by are deep valleys buried beneath the glacial drift.

The peculiar conditions which permit the development of tidal power here are the existence of the funnel-shaped Bay of Fundy, which is the cause of the high tides, and the existence of two large, landlocked bays, Passamaquoddy and Cobscook, which serve as basins for the tidal power scheme. The Bay of Fundy is probably due in part to faulting, in part to stream erosion and glacial erosion. Passamaquoddy and Cobscook bays are due principally to stream erosion modified by glacial erosion, but the shape of the bays was controlled by the complex structure of the rocks.

This is practically the only place on the east coast of the United States where this peculiar combination of high tides and landlocked bays makes the development of tidal power possible. Higher tides occur farther up the Bay of Fundy in Canada, but not in the United States.

#### TIDAL DAMS

To close off Cobscook Bay from Passamaquoddy Bay and the Bay of Fundy requires six dams, two large and four small ones. (Fig. 2.)

\* Ernst Antevs, "Late Quaternary Changes of Level in Maine." *American Journal of Science*, 5th Series, Vol. 15, pp. 319-336, April, 1926. Irving B. Crosby and Richard Lougee, "Glacial Marginal Shores and Marine Limit in Massachusetts." *Bulletin of the Geological Society of America*, Vol. 45, pp. 441-462, 1934.





FIG. 2. — MAP OF COBSCOOK BAY AND VICINITY, SHOWING TIDAL DAMS AND HAYCOCK HARBOR PUMPED STORAGE RESERVOIR

Eastport is on Moose Island which separates Cobscook and Passamaquoddy bays, and which is connected with the mainland by highway and railroad bridges. The railroad crosses from the mainland by two small islands, and three small dams are planned across the openings. These are known as the Carlow Island Dam and the Pleasant Point Dams. They have been built as rock fills and would be made tight by an impervious blanket on the Cobscook Bay side.

The other three dams will connect the southern point of Moose Island with Lubec Neck. In between are Treat and Dudley islands, and the Eastport Dam, 3,400 feet long, would extend from Moose Island to Treat Island, the short Treat-Dudley Dam, already built, would connect these two islands, and the Lubec Dam, 3,500 feet long, would extend from Dudley Island to the mainland at Lubec. The principal problems are concerned with the two large dams. The lock would be in the north end of Treat Island and the filling gates in the south end of this island.

The basins now occupied by Cobscook and Passamaquoddy bays were made by preglacial streams at a time when the land stood several hundred feet higher than now. The Passamaquoddy basin was drained through a deep valley which extended southerly along the International Boundary to the vicinity of Eastport and then apparently turned northeasterly and emptied into the Bay of Fundy valley. The Cobscook Basin was drained through a valley which passed south of the Eastport hills then turned northeast and joined the Passamaquoddy valley. The preglacial Cobscook and Passamaquoddy streams not only eroded the basins which made this project possible, but they also eroded the gorges draining these basins, and thus made the most serious problem for the construction of this project. The gorge draining the Passamaquoddy basin is very deep; one sounding showed 390 feet of water, and it is not known how much farther it is to bedrock. It is possible that this gorge was deepened by glacial erosion. If the International project were carried out it would be necessary to close this gorge with a rock-fill dam.

The gorge leading from the Cobscook Basin was made by a smaller stream, has probably not been deepened by glacial erosion, and is not so deep. The deepest water at the site of the Eastport Dam is 115 feet, but one boring showed 283 feet to rock, and it may be a little deeper. (Fig. 3.) The maximum thickness of overburden is about 170 feet, of which the upper 110 feet consist of silty clay with scattered thin beds of fine sand, and the lower 60 feet consist of clay with more numerous and thicker beds of sand.

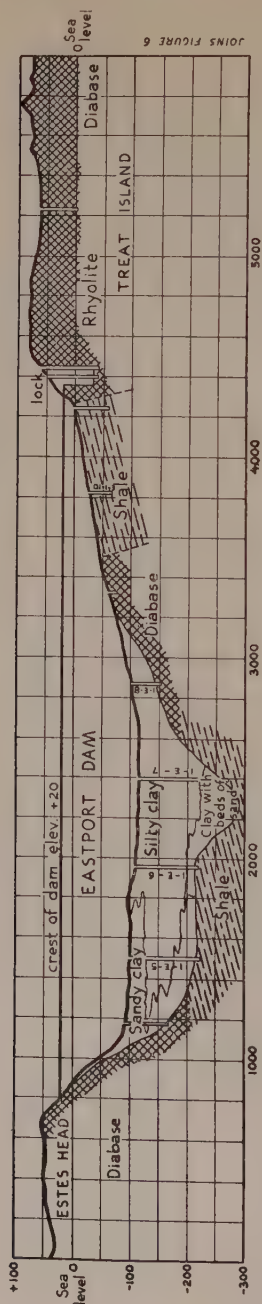


FIG. 3. — GEOLOGIC SECTION OF EASTPORT DAM SITE ALONG AXIS OF DAM



Deep water borings were made from two large lighters, each of which was anchored to six 7-ton anchors by lines to winches which permitted movement of about 1,500 feet in any direction without shifting the anchors. (Fig. 4.) Each boat had an outrigger on the side for the support of a huge spud which consisted of a steel casing 21½ inches in diameter. This spud had a pad 7 feet in diameter at the bottom and a removable platform at the upper end. The spud was kept vertical by tackles from the outrigger. Six-inch casing was driven inside the spud to rock, and a core boring was made in rock. Five-inch samples were taken of the overburden. These borings were made in currents as great as 6 knots, and with a maximum tidal range of 26 feet.

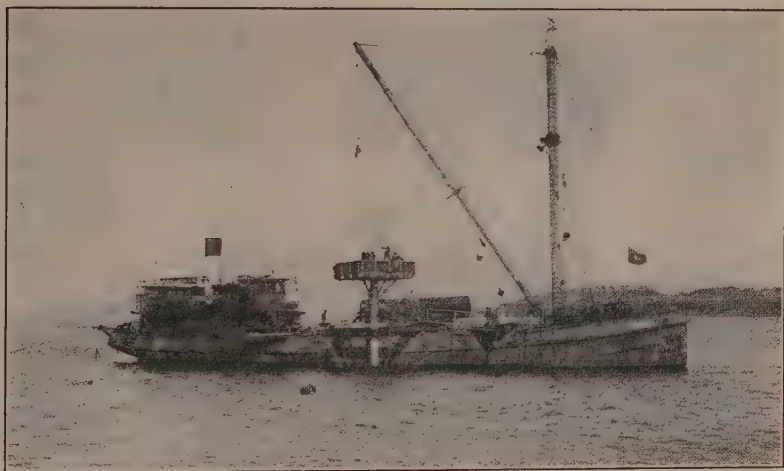


FIG. 4. — DEEP WATER BORING RIG

(From Communication No. 3, Second Congress on Large Dams, Washington, D. C., 1936)

Geologic sections were prepared from the scanty borings, using geologic principles of the erosion of these valleys, and the mode of deposition of these deposits to interpolate between the borings. The soil samples were tested in the soil mechanics laboratory for consolidation, shear, water content, grain size, etc. The shearing strength of the clay samples generally ranged from a little less than 0.2 ton per square foot, and the water content varied from 25 to 35 per cent of the solids. From the geological and laboratory information, estimates were made of the probable settlement of the dam.

On account of hydraulic conditions it was decided to make the dam 225 feet wide at elevation —30 mean sea level. The necessary

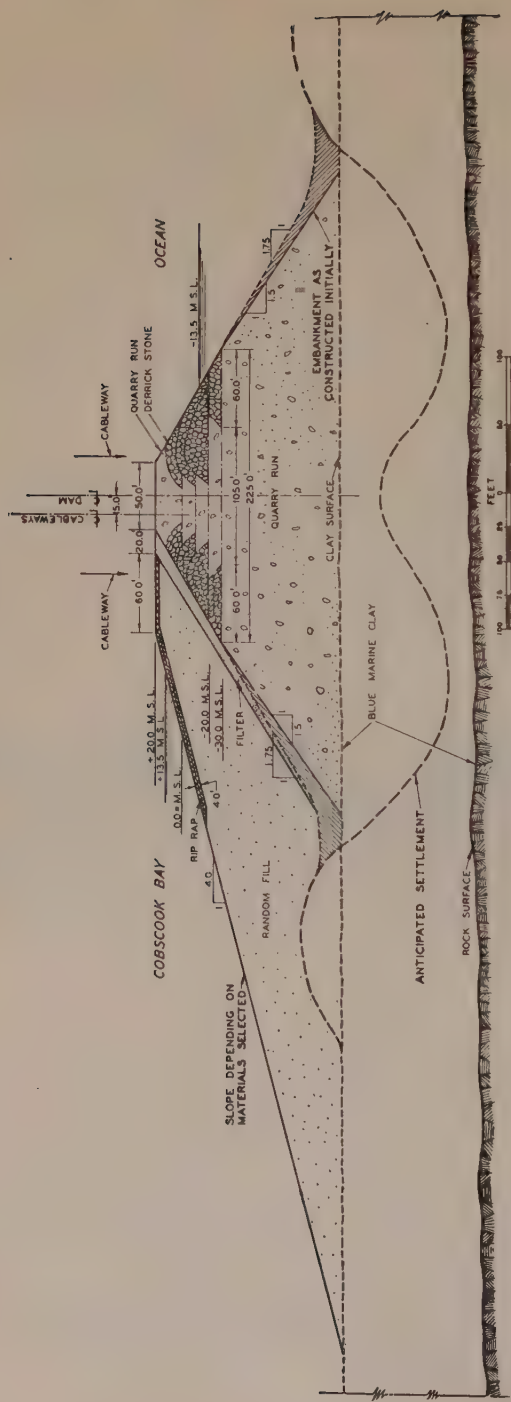


FIG. 5. — SECTION OF ROCK-FILL DAM SHOWING ANTICIPATED SETTLEMENT

(From Communication No. 3, Second Congress on Large Dams, Washington, D. C., 1936. Paper by Hugh S. Casey, Captain, Corps of Engineers, United States)

slopes of 1 on  $1\frac{1}{2}$  to 1 on  $1\frac{3}{4}$  resulted in a base width up to 500 feet for the rock-fill dam. (Fig. 5.) In order to determine the nature and extent of stress in the plastic clay under such loading, photo-elastic gelatin tests were run in the soils laboratory, and these were confirmed by model tests on clay. These tests indicated an outward flow of the clay with settlement of nearly 100 per cent under both toes and approximately 50 per cent under the center of the dam.\* It would have been possible to design the dams with flat slopes, 1 on 5 to 1 on 10, but this would have required much more rock than would be necessary for 100 per cent settlement. Therefore, contrary to normal practice, the design contemplates failure of the plastic clay foundation as the most economic solution.

At the site of the Lubec Dam the water is not so deep and the thickness of the overburden is not so great. (Fig. 6.) Geologic studies indicated that there was a depression in the bedrock surface near the northern end of this dam which was not found by the borings. This depression is due to a fault zone which crosses the line of the dam. It was found that by moving the dam a short distance to the east a better bedrock profile with less thickness of overburden could be obtained. The maximum thickness of overburden is about 135 feet, of which the upper 90 feet are silty clay with the scattered thin beds of fine sand, and the lower part is clay with more numerous and thicker beds of sand. The conditions here are, therefore, similar to those at the Eastport Dam, but are not quite so severe. The Lubec and Eastport dams will together require approximately 7,000,000 cubic yards of rock.

The power house is located at a narrow neck connecting the two parts of Moose Island. The neck is composed of clay, silt and sand. Numerous borings and jettings were made on the site of the power house and the headrace and tailrace channels. From this data and from geologic studies a contour map of the bedrock surface was prepared. This showed that but little rock excavation would be necessary for the headrace and tailrace channels. The rocks upon which the power house would rest are shale, slate, sandstone and rhyolite (felsite). The shale is thick bedded and is amply strong to sustain the load of the power house, and the other rocks are still stronger.

It would seem at first that rock for the rock-fill dams could be obtained almost anywhere from the numerous diabase and rhyolite ledges, but careful observation shows that much of the rock is so closely cut by fractures and incipient fractures that it is not possible to obtain

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\* Hugh J. Casey, "Construction of Rock-Filled Dams in Flowing Water on the Passamaquoddy Tidal Power Project." Second Congress on Large Dams, Communication No. 3, 1936.



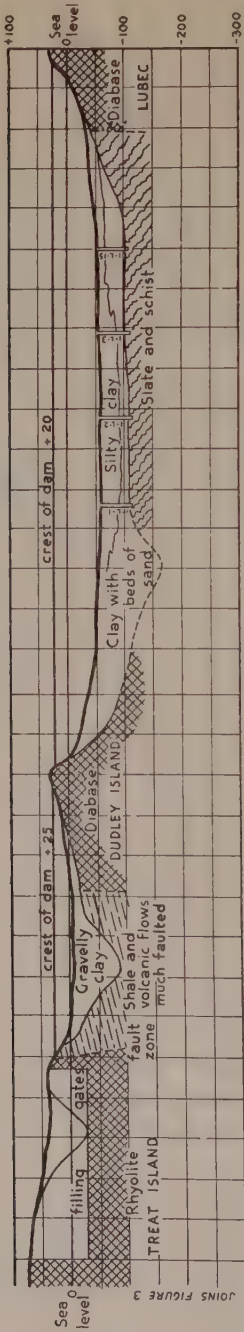


FIG. 6. — GEOLOGIC SECTION OF LUBEC DAM SITE ALONG AXIS OF DAM

large blocks. Shackford Head offered a very convenient quarry site, but here the rock was unusually badly fractured. Rock from here would, however, be suitable for the bulk of the rock fill and for aggregate, and it was planned to open a quarry here. Excavations for the lock and filling gates on Treat Island would provide a large amount of rock suitable for rock fill except where large blocks were needed. At Black Head, on a branch of Cobscook Bay, the diabase is more massive, and it is probable that 4-foot blocks could be obtained. If blocks of the necessary size were not available at any of these places they could be obtained about twenty miles north of Eastport, where there are numerous ledges of massive granite near the shore of Passamaquoddy Bay. Here large blocks of any desired size could be quarried and transported by water to the dams.

#### HAYCOCK HARBOR PUMPED STORAGE RESERVOIR

The site of the Haycock Harbor Pumped Storage Reservoir is located among low hills about twelve miles south of Eastport and on the south side of Cobscook Bay. (Fig. 2.) The reservoir is planned for a flow line at an elevation of 120 feet, and would occupy parts of the drainage basins of three small streams. The natural drainage into this basin would be negligible, and it would be operated as a pumped storage plant, using excess power at the peak of the tidal power cycle to pump ocean water up into this reservoir. When the tidal power plant was not running this stored water would be used in the power house of the pumped storage plant to generate power, thus maintaining a steady supply of power from the Quoddy project.

It was at first planned to have the pumping and power plant at Haycock Harbor and pump directly from the ocean into the reservoir. When the Haycock Harbor site was first chosen by Cooper it was expected to use Cobscook Bay as a low-level pool. Since then it was decided to make the bay a high-level pool, and it was also found that the necessary layout of dams to obtain a power house in a satisfactory location on Haycock Harbor would be more expensive than expected, and therefore a study was made for a power house location on the Whiting-Lubec road, with the pumping plant taking water from Cobscook Bay at near high-tide level which would result in an operating economy. It was also found that this power house location with the resulting dams had advantages over the original location on Haycock Harbor, and this was the preferred plan at the time investigation of this part of the project was stopped. Haycock Harbor Reservoir

would require twelve dams, large and small, but the study of this project involved investigation of ten additional dam sites.

These dam sites are of two types: (1) those across old stream valleys, now partly filled with glacial and marine deposits, and (2) those across saddles and along ridges. Each type has practical problems peculiar to it, and recognition of the type to which a particular site belongs aids in understanding the problems of the site. As regards the valley type dam site the valleys are partially filled with glacial deposits in every place known. Even where the stream is now flowing over ledge a buried valley is believed to exist near by. The buried valley may not be indicated by any surface stream. The filling of these buried valleys is often pervious and may allow seepage under or around the dam. Where dams cross clay-filled valleys the possibility of settlement must be studied. Buried valleys are known to lead from the reservoir in four places and four of the dam sites chosen and several of the alternate sites are of the valley type. The saddle-type dam sites cross saddles between rock hills, and in places follow ridges. Rock is often at or near the surface, and the overburden is generally not clay, but there are some exceptions. In at least two cases the saddles are deeply buried, and there is a seepage problem at one and a settlement problem at the other. In the case of dams on rock ridges the tightness of the rock becomes a factor.

Buried valleys are very common throughout New England and the adjacent glaciated regions, and the valley type of dam site is most common. There is, however, a very important difference between the dam sites on this project and dam sites in other parts of New England, in that this area has been submerged beneath the sea in post glacial time, and the rocks and glacial deposits are partially blanketed by marine deposits, largely clay. This increases the difficulty of investigating the sites and increases the problems of the sites. Saddle type dam sites are less common elsewhere but exist in connection with some reservoirs. The dams at such places are usually small earth dikes, and it is seldom that they have the problems of some of the saddle type dam sites of the Passamaquoddy project.

The Haycock Harbor Reservoir may be divided geologically along its northeast-southwest axis. The rocks to the northwest are rhyolite tuffs and flows, with some small areas of diabase tuffs and flows and of shale. To the southeast is an extensive mass of intrusive diabase in which are elongated areas of the Quoddy formation. This last is the oldest formation of the Eastport region, and consists of shale, quartzose shale, slate and quartzite, with occasional flows or tuff beds of rhyolite



or diabase. It is an extremely variable formation formed of sediments, lava flows and volcanic ash, and is less resistant to erosion than the diabase. As a result, valleys tend to lie in the Quoddy formation, and it is of greater practical importance in relation to the dams than its area would indicate. Three shear zones cross the reservoir, one of which separates the rhyolite from the intrusive diabase. One of these zones intersects a dam site, but only a small earth dike was planned here, and the presence of the shear zone would have no effect on the plans.

The most interesting problems were concerned with the location of the power house and necessary dams at Haycock Harbor, and with the dam which took the place of these when the power house location was removed to the Cobscook Bay side of the reservoir. Haycock Harbor is a long narrow inlet eroded by a stream in a belt of shale of the Quoddy formation at a time when the sea stood at a lower level. This inlet is bordered north and south, and at its head by diabase, which is a much more resistant rock. Wiggins Brook flows into the head of the inlet falling over diabase ledges. The original plan for a dam at Haycock Harbor contemplated crossing the valley of Wiggins Brook, between two rock hills. Early in the geological study of this site, before any borings had been made, evidence was found of a buried valley north of the present channel of the brook. The fact that the brook valley was wider above the place where it crossed the diabase ledges into tide-water, and that just north of this a belt of less resistant shale of the Quoddy formation leads to tidewater, indicated the probable existence of a buried valley in the bedrock surface north of the present stream and north of Haycock Harbor. Other evidence indicated that this valley was probably filled with clay; therefore two borings were located in the probable location of this valley. One of these borings reached rock near sea level, the other did not encounter rock until an elevation of 34 feet below mean sea level was reached. It is not certain that this boring was in the deepest part of the valley, but it was evident from the study of the surface outcrops that the buried valley was narrow and gorge-like. It was also found that the filling of this buried valley was soft clay which would require a broader base and flatter slopes than would otherwise be used for the earth dam.

Three different sites for the power house and six different dam sites were studied, but it was found that whatever arrangement of power house and dams was chosen it would be necessary to cross this buried clay-filled valley. There were, however, no other serious problems here, and there would be little seepage under or around the dams.

When it was decided to locate the power house on the Cobscook Bay side of the reservoir, a dam site was chosen northwest of Haycock Harbor which took the place of those in the vicinity of the harbor and had certain advantages. This new site, known as Site F, also crossed the valley of Wiggins Brook and the inland extension of the buried valley. The geological indications were that the buried valley would be much less deep at this point, due to the fact that the belt of shale did not extend so far inland, and that the preglacial stream had not eroded so deeply in the more resistant diabase. This indication was borne out by the borings. There is less clay at this site and less probability of settlement of the dam. This dam would be the largest in the Haycock Harbor Reservoir project, and would require approximately 2,670,000 cubic yards of earth.

Dam site 7, on the northwest side of the reservoir, extends across saddles and along ridges for a distance of 6,300 feet. The southern part of the site is across a broad saddle where rock is buried 50 feet deep. The overburden is sand, and an impervious blanket on the reservoir side of the dam would be necessary to reduce seepage under the dam. This dam would require approximately 1,082,000 cubic yards of earth. Part of this dam would be on a narrow rock ridge of rhyolite. The tightness of this rock was tested by water pressure in an inclined boring. It was satisfactorily tight, and a masonry structure on the rock ridge would probably have been used here.

Dam site 8 is at the power house site on the west side of the reservoir. It crosses two small buried valleys in which there is sand buried beneath a layer of clay. The clay will effectively prevent seepage under the dam. However, the power house site would be on diabase near a contact with shale. Some of the rock is deeply weathered, and treatment of the foundations would probably be necessary. This is practically the only place on the reservoir project where the condition of the rock is an important factor, and it is not a serious difficulty here.

Dam 12 would be a long low dam across a broad sandy saddle between two sluggish streams. There was physiographic evidence of a buried valley under this saddle, and its existence was proved by a boring which failed to reach rock at a depth of 90 feet.

The twelve dams would require approximately 5,700,000 cubic yards of earth, and the location of suitable deposits was a difficult problem. Generally in a glaciated region suitable materials for earth dams are not lacking, but, due to the submergence of the glacial deposits here, much of the boulder clay was washed by the sea which

removed the fines and left sand and gravel. In addition much of the area was covered by a blanket of marine clay which probably conceals many of the remaining deposits of boulder clay.

A careful physiographic study of the area was made, both by map and in the field, and a number of likely looking places were located, many of which proved to have usable material. The investigation for borrow material was not complete when this part of the project was stopped, but it appeared that sufficient material could be obtained, though it would be necessary to use many small deposits.

The proposed Haycock Harbor Reservoir encountered no very serious difficulties, but there were several factors which tended to increase the cost. Several of the dams would cross valleys filled with marine clay, and flatter slopes with greater yardage would be necessary. Probably the most serious factor is the scarcity of large deposits of suitable embankment material which could be worked economically. These factors caused the investigation of another reservoir site nineteen miles north of the tidal power house site at Eastport and just south of Calais.

Before leaving consideration of the Haycock Harbor Reservoir it should be added that there is no danger of contamination of public water supplies by salt water from the reservoir. The Lubec supply is several miles distant, and is separated by several valleys and rock hills from the reservoir.

#### THE CALAIS PUMPED STORAGE RESERVOIR

The Calais Pumped Storage Reservoir would utilize natural basins in the hills southeast of Calais, Maine. (Fig. 7.) These basins are now occupied by several lakes and ponds, the three larger of which had been raised by small dams years ago when the granite industry was active here. The reservoir could be of greater or lesser size, as required. The southern lake basin was separated by a low ridge through which it was planned to cut a canal if this additional storage were needed. There were many alternate dam sites, and arrangements were possible which would give greater or less storage, but final choice had not been made when this investigation was stopped. Four locations of the power house were possible, three of which would require tunnels, but final selection had not been made. A flow line of 198 feet was considered, but a flow line a little higher or considerably lower was feasible. A minimum of seven dams would be necessary if the southern basin were not used, but twenty-eight dam sites were studied.



The rocks of this area are all granitic and include true granites, norite and quartz diorite. The granites are gray, pink and red, and the last two are very beautiful stones. The red granite takes a high polish and is excellent for ornamental work. The gray granite is a very

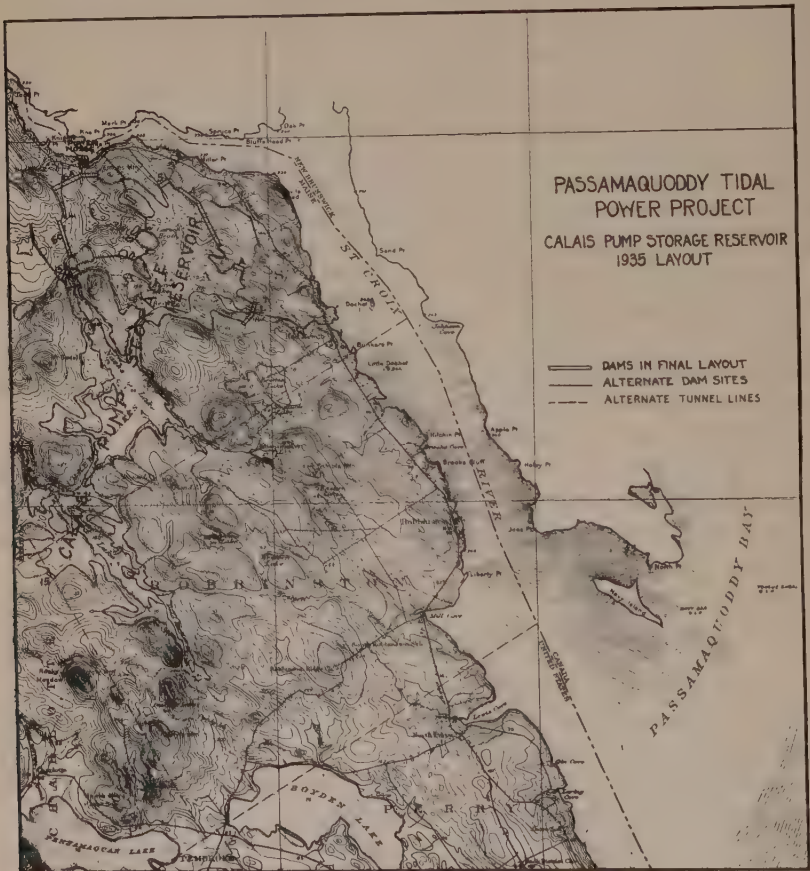


FIG. 7.— MAP OF CALAIS — AREA SHOWING CALAIS PUMPED STORAGE RESERVOIR

massive rock and huge blocks can be obtained from it. The norite and quartz diorite are very dark gray, almost black, and are quarried as "black granite." This stone takes a very high polish, but cut surfaces are very light in comparison to the very dark polished surface. It is excellent monumental stone for inscriptions, and is still quarried in a

crude way on a small scale. The decadence of this granite industry appears to be partly due to lack of initiative. There are few places where such a variety of beautiful granites can be obtained. All of these granites are excellent foundation rocks, and cause no problems for this project.

Although the lower part of this area was covered by the late glacial sea, the effects of the submergence are not as pronounced as in the Haycock Harbor area. Much less of the boulder clay was removed by wave action and much less marine clay blankets the glacial deposits. Therefore the dam sites have more favorable conditions, and large deposits of boulder clay suitable for earth dam construction are available.

These dam sites are of two types: valley sites and saddle sites, as in the Haycock Harbor area. The stream valleys are partly filled with glacial deposits, but so far as is known these are nowhere as thick as at some of the Haycock Harbor sites. Some of the buried valleys are not now followed by streams, and the possibility of seepage from the reservoir through these was studied but did not appear to be serious. Five dam sites are certainly of the valley type, but most of the dam sites are of the saddle type and many of these have excellent foundation conditions. At some of these sites the basin on the inner side is filled with bog material, and the bog overflows, giving the appearance of a valley type dam site, but there is the important difference that there is no buried valley, and rock is near the surface.

The largest dam would be 6A, at the northern end of the reservoir. The height from the brook would be 126 feet and the length 3,500 feet. From the dam short penstocks would lead to the power house on tidewater. It would be possible to avoid this large dam by building several small ones, but this would necessitate a long tunnel to the power house. Final choice between these alternatives had not been made when this part of the project was stopped. This site is of the valley type, but the valley filling is only 30 feet thick, is impervious, and is partly boulder clay, which provides good foundation conditions for an earth dam.

Dam site 13A is a typical example of the valley type. (Fig. 8.) Physiographic considerations caused me to suspect a buried valley here, and a boring on the bank of the stream found rock 48 feet below the level of the stream. The overburden consisted of 26 feet of peat over 22 feet of clay and sand. Removal of the peat would be necessary here, and although there would be no seepage problem there might be some settlement.

Dam site 6C crosses a bog where the depth to rock is not great, passes over a low rock ridge, and crosses a saddle between rock hills. It appears at first to be a saddle type dam site, but physiographic study shows that the buried valley which was proved at dam site 13A must pass under this saddle. A boring reached rock at 32 feet, but it is probable that there is a narrow gorge some 20 feet deeper, and another boring was recommended but had not yet been drilled when work was stopped.

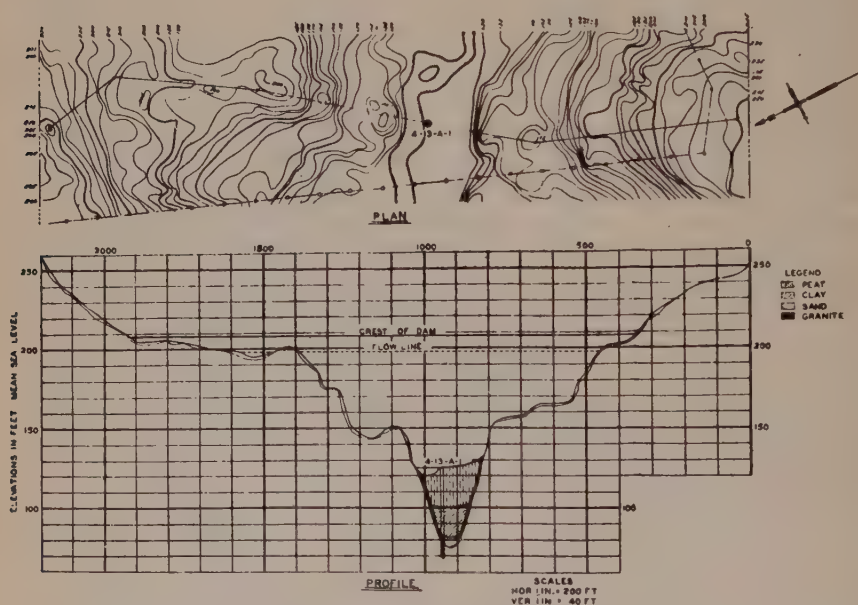


FIG. 8. — MAP AND GEOLOGIC SECTION OF DAM SITE 13A

A typical valley type dam site

Dam site 5 is a typical saddle type site with satisfactory conditions for an earth dam. (Fig. 9.) At the surface in the bottom of the saddle are about 6 feet of silty clay beneath which are some 20 feet of boulder clay and then granitic bedrock.

Several of the possible arrangements of dams and power house would require tunnels to the power house. In every case the rock is excellent for tunnelling.

In respect to embankment material this project is especially fortunate. Northwest of dam site 6A is a broad, flat-topped hill capped with a thick deposit of boulder clay. This is believed to contain at



least 5,000,000 cubic yards of usable material. Just west of this is a similar hill on which over a million cubic yards of boulder clay was proved and which probably has much more. In addition, smaller deposits of boulder clay are scattered about the reservoir area, and there are deposits of sandy material suitable for the outer sections of the dams. It would therefore be possible to obtain suitable embankment material very economically.

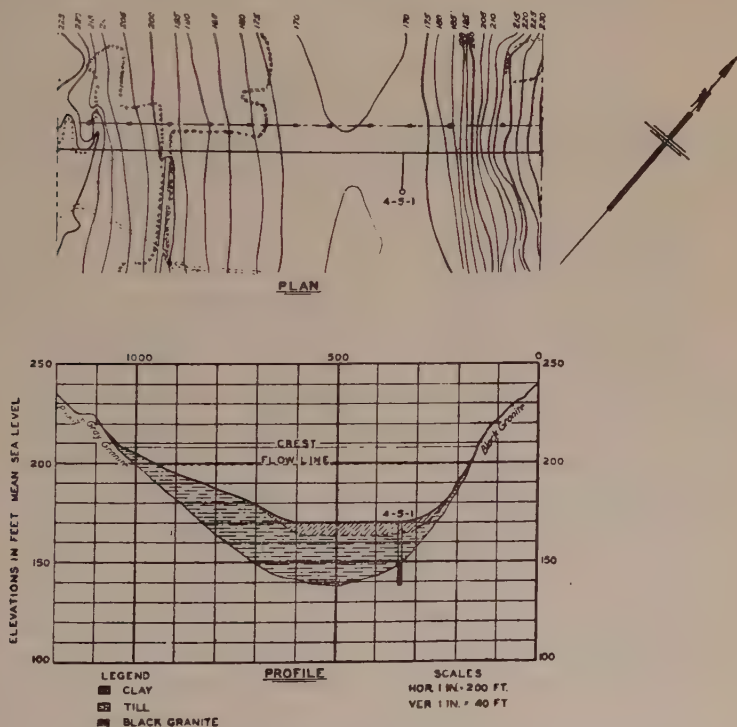


FIG. 9. — MAP AND GEOLOGIC SECTION OF DAM SITE 5

A typical saddle type dam site

Although the Calais Storage Reservoir project is not ideal in every way, it has no serious geological difficulties and is superior to the Haycock Harbor Reservoir project as regards settlement of dams, seepage under dams, and availability of embankment material. This reservoir project has no serious geological difficulties and has many advantages. An unusually tight reservoir could be obtained and conditions are conducive to economical construction.

## MODEL ANALYSIS OF STRUCTURES

BY DR. JOHN B. WILBUR, MEMBER \*

(Presented at a meeting of the Designers Section of the Boston Society of Civil Engineers held on December 8, 1937)

### INTRODUCTION

THE investigation of structural action by means of models presents a method of attack which under some circumstances may be of considerable value to structural engineers. This by no means implies that model testing is in general preferable to analytical methods, nor that the model approach is one easily carried out by those untrained in the theory of stresses or unskilled in the technique of laboratory procedure. The proper sphere of problems to which the model analysis approach should be considered is difficult to define, and is a function, among other things, of our knowledge of model procedure. Structural analysis by means of model studies is a comparatively new development, and one in which there is every reason to believe that rapid strides will be made. New developments will open up new fields of application.

It is, however, possible to outline in a general way the types of problems which at the present time may be suitably approached by model study. In complicated structures, particularly in rigid frame structures which are highly indeterminate, moments, shears, direct stresses and deflections may be determined. Under these circumstances the model analysis may be used to obtain an independent check of an analytical solution, or it may be counted upon to furnish by itself the data necessary for design, provided the methods and techniques used have been thoroughly tested in previous work. Even where the total direct stresses, moments and shears acting on a member may be easily determined by analytical methods, considerable uncertainty may exist as to the actual stress distribution in a member. Here the problem may become so theoretically complicated that a model study appears to be the only workable approach to a substantially correct solution. The study of the properties of materials, while necessarily interrelated with model analysis, does not strictly constitute a part of the field of model analysis.

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Stress analysis by means of models may be subdivided into two general approaches: direct methods and indirect methods. In the direct methods loads are applied to the model, which are comparable to the actual loads which would be applied to the structure reproduced, and the resulting deformations so interpreted that the stresses are determined. In the indirect methods, the applied loads bear no direct connection to actual loads which would be applied to the real structure, but they are, nevertheless, such that the structural action of the model may be interpreted to give the desired results.

### DIRECT METHODS OF MODEL ANALYSIS

The most obvious direct method of model analysis consists of measuring linear strains on a model with strain gages. The Berry

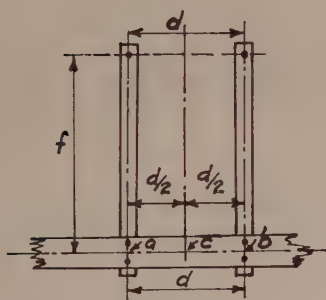


FIG. 1a  
(Before Distortion)

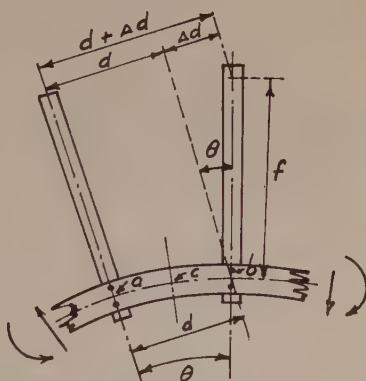


FIG. 1b  
(After Distortion)

strain gage and the Huggenberger tensometer are two well-known gages which may be used for this purpose. With these and similar gages, direct stresses in members may be determined, and it is believed that methods might be developed whereby they could also be used in the determination of bending moments and shears.

The determination of bending moments in members of rigid frames is of particular interest in model analysis. Any number of direct approaches might be made in their determination, two of which will be discussed briefly. In theory, the simplest set-up would consist of measuring the change of slope between two points  $a$  and  $b$  as shown in Fig. 1.



Then  $\theta = \frac{\Delta d}{f} = \text{area under } \frac{M}{EI} \text{ curve between } a \text{ and } b \text{ by the first moment area theorem.}$  Since shear between  $a$  and  $b$  is constant, the moment curve between the two points is a straight line. Hence, if  $I$  and  $E$  are constant,  $\theta = \frac{\Delta d}{f} = \frac{1}{2} \left\{ \frac{M_a}{EI} + \frac{M_b}{EI} \right\} d = \frac{M_c}{EI} d$ . From this expression,  $M_c = \frac{EI \Delta d}{fd}$ . The term  $\Delta d$  may be determined by reading the distance between points on the ends of the arms with a microscope before and after the distortion of the beam occurs; once  $\Delta d$  is known,  $M_c$  may be calculated, provided the  $E$  of the material used is also known.  $I$ ,  $f$  and  $d$  are, of course, easily measured. By determining the moments at two points in a beam, the shear in the beam can be computed.

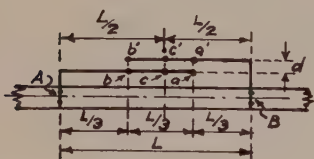


FIG. 2a  
(Before Distortion)



FIG. 2b  
(After Distortion)

While simple in theory, various factors contribute to making accurate determinations of moments by the method just described extremely difficult. The M. I. T. moment indicator, developed in principle by Prof. A. C. Ruge, and adapted to the problem at hand by Mr. E. O. Schmidt, possesses inherent advantages which lead to a more accurate determination of moments. This indicator is based in theory upon the slope deflection equations, and may be explained by reference to Fig. 2.

$$\text{From Fig. 2b, } bb' = d + \frac{L}{3} \theta_A + \frac{2L}{3} \theta_B$$

$$\therefore bb' - d = \text{relative movement of points } b \text{ and } b' = \underline{b} =$$

$$\frac{L}{3} \theta_A + \frac{2L}{3} \theta_B = \frac{L}{3} (2\theta_B + \theta_A); \text{ also } aa' = d + \frac{2L}{3} \theta_A + \frac{L}{3} \theta_B$$

$\therefore aa' - d =$  relative movement of points  $a$  and  $a' = \underline{a} =$

$$\frac{2L}{3} \theta_A + \frac{L}{3} \theta_b = \frac{L}{3} (2\theta_A + \theta_B); \text{ also } cc' = d + \frac{L}{2} \theta_a + \frac{L}{2} \theta_b$$

$\therefore cc' - d =$  relative movement of points  $c$  and  $c' = \underline{c} =$

$$\frac{L}{2} \theta_a + \frac{L}{2} \theta_b = \frac{L}{2} (\theta_A + \theta_B)$$

By the slope deflection theorem —

$$M_{AB} = \frac{2EI}{L} (2\theta_A + \theta_B) = \frac{2EI}{L} \cdot \frac{3a}{L} = \frac{6EIa}{L^2}$$

$$M_{BA} = \frac{2EI}{L} (2\theta_B + \theta_A) = \frac{2EI}{L} \cdot \frac{3b}{L} = \frac{6EIb}{L^2}$$

$$V_{BA} = \frac{M_{AB} + M_{BA}}{L} = \frac{1}{L} \left[ \frac{2EI}{L} (2\theta_A + \theta_B + \theta_A + 2\theta_B) \right] =$$

$$\frac{6EI}{L^2} (\theta_A + \theta_B) = \frac{6EI}{L^2} \cdot \frac{2c}{L} = \frac{12EIc}{L^3}$$

Hence, if  $a$ ,  $b$  and  $c$  are determined from the model study, by reading movements of points on the moment indicator with a microscope, and if  $E$  is known, the moments and shear in the member considered may be determined.

The material used in the model must behave elastically if correct results are to be obtained. It should have a relatively low value of  $E$ , so that the distortions will be large enough to be measured with a fair degree of accuracy. It should be easy to work with so that the construction of the model will not involve too much time and expense. Celluloid meets these requirements, and consequently is much used. Two difficulties of major importance are present, however, when celluloid is used. While celluloid has a definite value of  $E$  at a given instant, it changes appreciably from day to day, due to drying and consequent hardening and to changes in temperature. More serious still, celluloid creeps under load, that is, if a load is applied, while some 85 per cent of the deformation occurs within a few seconds, the remaining 15 per cent occurs more slowly. It would be necessary to wait an appreciable period — in the neighborhood of fifteen minutes — before motion would be essentially complete. Even then small movements would still be

occurring. So many measurements must be made that to wait for the creep to occur on each measurement would be impractical. Fortunately the strains due to creep are proportional to the fiber stresses acting, as are also the strains due to the initial elastic character of the celluloid. It is therefore correct to say that the effect of creep corresponds to a lowering of the modulus of elasticity. At any instant, however, the effective  $E$  for the entire model is constant. Referring to Fig. 3a, if load  $P$  is applied at the end of the cantilever shown, the elastic curve will progressively assume different positions, and readings on a moment indicator would change correspondingly. Suppose, however, the end of the cantilever is subjected to a fixed deformation as shown in Fig. 3b. Then the cantilever end remains in a fixed position, and the elastic curve of the beam does likewise. Although  $E$  is changing with time, the shape of the elastic curve does not depend upon the

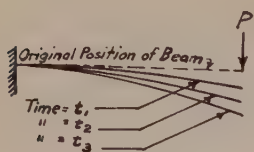


FIG. 3a

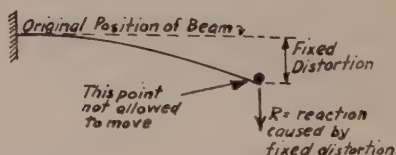


FIG. 3b

particular value of  $E$  at any instant, as long as that value is constant for the entire beam. A definite reading on the moment indicator may be made. The value of  $R$  decreases with time because of creep, but the reading on the moment indicator corresponds to that which would result if an unknown force,  $P$ , which remained constant, acted at the end of a cantilever beam of unknown but unchanging  $E$ . Under these conditions, values of moment read on the indicator for different points on the beam would be in the correct ratio to each other; in other words, relative values of moment at points along the beam due to a load  $P$  at the end may be determined.

For these results to be of use, the actual value of the moments in terms of  $P$  must be obtained. Under most circumstances this may be carried out by applying an equation of statics which involves a number of bending moments, the relative value of each being known. For example, referring to Fig. 4, suppose the relative values of the moments in the column ends  $A$ ,  $B$ ,  $C$  and  $D$  due to load  $P$  have been determined



by the moment indicator. From these relative values the constants  $C_1$ ,  $C_2$  and  $C_3$  are known in the equations —

$$M_{BA} = C_1 M_{AB}$$

$$M_{CD} = C_2 M_{AB}$$

$$M_{DC} = C_3 M_{AB}$$

Then by statics

$$Ph + M_{AB} + M_{BA} + M_{CD} + M_{DC} = 0$$

$$Ph + M_{AB} (1 + C_1 + C_2 + C_3) = 0$$

$$M_{AB} = \frac{-Ph}{1 + C_1 + C_2 + C_3}$$

This involves reading the indicator at four points in order to determine the value of one moment. As a rule, however, all the moments in the frame are required, and they may now all be easily evaluated, so

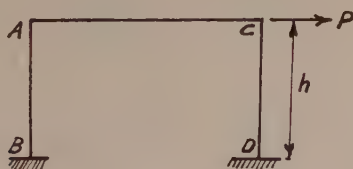


FIG. 4

that when an equation of statics is available, the fact that only relative values of moments can be determined by the method outlined is no great disadvantage.

In the determination of secondary moments in trusses, however, no equations of statics are available to convert relative values of moments to absolute values. Because of this difficulty, a method has been developed which can be used to obtain absolute values of moments directly with the moment indicator. Suppose on the cantilever beam shown in Fig. 5 a fixed deflection is imposed by moving  $B$  to  $B'$  and holding it there. The force required to impose this deflection is carried by the steel wire 1-2 to a celluloid spring balance 3-4-5-6 which is composed of two crossbeams 3-4 and 5-6 which are made from the same piece of celluloid as is the cantilever beam. The two crossbeams are connected by steel strips 3-5 and 4-6. From the crossbeam 5-6 the



which  $K_2$  has been determined. Readings  $e_1$  and  $e_2$  are read as before. The following relations then hold, where  $M$  is the required moment:

$$M = \frac{EI}{L^2} e_1$$

$$E = \frac{K_2 P}{e_2}$$

$$\therefore M = \frac{I}{L^2} \frac{K_2 P}{e_2} e_1 = K_3 \frac{e_1}{e_2} P$$

In the foregoing expression  $K_3$  is a constant which can be calculated, while  $e_1$  and  $e_2$  are relative movements read on the moment indicator and spring balance, respectively. Thus  $M$  becomes known as a direct function of  $P$ .

Another direct method of stress analysis with models is made possible by the use of polarized light, and is called the photo-elastic method. It is of particular importance in the solution of problems of stress distribution. It is mentioned in this paper for the sake of completeness, but will not be discussed further.

#### INDIRECT METHODS OF MODEL ANALYSIS

The best known indirect method of model analysis in this country is that which was developed by Prof. George E. Beggs of Princeton. Referring to Fig. 6, let  $\delta_{ab}$  be the downward deflection of point  $a$  due



FIG. 6

to a unit downward load at  $b$ , and  $\delta_{aa}$  be the downward deflection of point  $a$  due to a unit downward load at  $a$ .

If a reaction,  $R_a$ , existed at  $a$ , the deflection of  $a$  due to a unit downward load at  $b$  would equal zero. This fact could be mathematically expressed as follows:

$$\delta_{ab} + (-R_a) \delta_{aa} = 0$$

from which —

$$R_a = \frac{\delta_{ab}}{\delta_{aa}}$$



Suppose that it is desired to obtain the influence line for the left vertical reaction of the continuous beam shown in Fig. 7a, and that a model is built as shown in Fig. 7b, to which a unit load is applied as shown at the left end. Let  $\delta_{aa}$  and  $\delta_{ba}$  be measured.  $\delta_{ba}$  is the downward deflection of point  $b$  due to a unit downward load at point  $a$ . By Maxwell's theorem,  $\delta_{ba} = \delta_{ab}$ , where  $\delta_{ab}$  is defined in discussing Fig. 6. It follows that —

$$\frac{\delta_{ba}}{\delta_{aa}} = \frac{\delta_{ab}}{\delta_{aa}} = R_a \text{ due to a unit load at } b.$$

If the model were made of celluloid, the creep of the model would preclude the application of a unit load as just suggested. It is possible, however, to impose a fixed distortion at  $a$ . To do this requires the original application of an unknown force,  $Q$ . As the celluloid creeps,

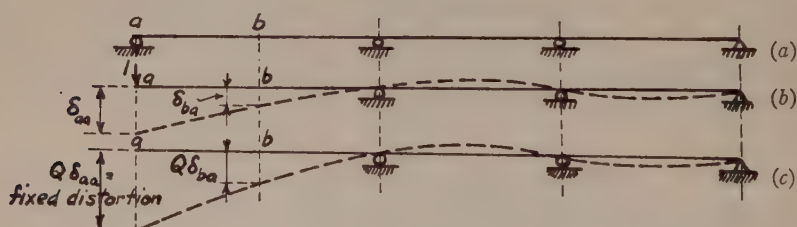


FIG. 7

the force  $Q$  changes, but the shape of the elastic curve does not, since the latter does not depend upon the particular value of  $E$  existing, as long as  $E$  is constant for the entire model; in other words, the model deflects exactly as it would if it had an unchanging although unknown value of  $E$ , and were acted upon by an unchanging but unknown force,  $Q$ . Under these circumstances the fixed distortion imposed at  $a$  may be interpreted as  $Q\delta_{aa}$ , and the distortion read at  $b$  as  $Q\delta_{ba}$ . It follows that the ratio of the distortion read at  $b$  to the distortion imposed at  $a$  is given by  $\frac{Q\delta_{ba}}{Q\delta_{aa}} = \frac{\delta_{ba}}{\delta_{aa}} = \frac{\delta_{ab}}{\delta_{aa}} = R_a$ , the reaction at  $a$  due to a unit load at  $b$ .

Inasmuch as  $b$  is any point on the beam, it follows that the ordinates to the influence line for  $R_a$  vary directly with  $\delta_{ab} = \delta_{ba}$ . Thus when a vertical distortion is introduced at  $a$ , the model takes the shape of the influence line for  $R_a$ . The true values of the ordinates are obtained by

dividing the resulting movements at various points by the magnitude of the imposed distortion.

The foregoing discussion has been limited to the special case of the determination of vertical reactions in order to simplify the presentation. The Beggs method is, however, applicable to the determination of any type of reaction or to internal stresses. If, for example, it is

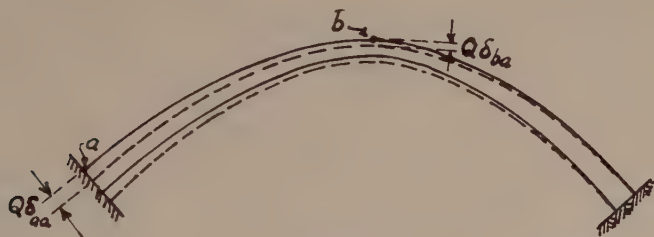


FIG. 8

desired to obtain values of the shear at the left reaction of the arch shown in Fig. 8, a displacement  $Q\delta_{aa}$  is introduced at that point which is in the direction of the required shear, that is, normal to the axis of the arch at that point. No tangential movement or change of slope at  $a$  is permitted. The ratio  $Q\delta_{ba}$  over  $Q\delta_{aa}$  then equals the shear at  $a$ , due to a unit downward load at  $b$ .

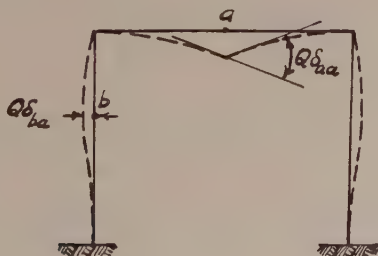


FIG. 9

When an internal stress is desired, it becomes necessary to cut the model in order that the deformation may be imposed. Suppose it is desired to obtain values of moment at the center of the girder of the building frame shown in Fig. 9. The model is cut at  $a$  and an angular distortion,  $Q\delta_{aa}$ , introduced. The two cut ends are not permitted any horizontal or vertical movement relative to each other. The ratio

$Q\delta_{ba}$  over  $Q\delta_{aa}$  then equals the moment at  $a$ , due to a unit horizontal load at  $b$ , inasmuch as  $Q\delta_{ba}$  is measured horizontally.

The necessity of cutting the model in order to obtain internal stresses seems to be a disadvantage of the Beggs method. The use of the moment indicator, which is simply clamped to the member where the moment is to be measured, has an advantage in this respect. The Beggs method has the advantage, however, that it gives stresses as a function of applied loads with the necessity of the spring balance.

### SIMULTANEOUS CALCULATOR

One of the major difficulties which is present in the analytical solutions for stresses in indeterminate structures is in the solution of simultaneous equations. In running tests on models, particularly in trying to develop new methods and techniques of model analysis, it is necessary to make analytical solutions as a check on the model work. For the solution of simultaneous equations there has been developed at Massachusetts Institute of Technology the simultaneous calculator, which under some conditions saves considerable time. The simultaneous calculator can, in a direct solution, solve nine or fewer linear simultaneous equations, and, by an indirect method, handle a greater number of unknowns. Arbitrary accuracy of results may be obtained by successive approximations. A description of this machine appeared in the December, 1936, issue of the Proceedings of the Journal of the Franklin Institute, which describes the theory upon which it is based.

### CONCLUSION

This description of model analysis has of necessity covered only some of the high spots in the theory of model analysis. It is hoped, however, that it will serve to make the inspection of the laboratory of greater interest than would otherwise have been the case.

# OF GENERAL INTEREST

## PROCEEDINGS OF THE SOCIETY

### MINUTES OF MEETINGS

#### Boston Society of Civil Engineers

NOVEMBER 17, 1937. — A regular meeting of the Boston Society of Civil Engineers was held this evening at Chipman Hall, Tremont Temple, and was called to order by the President, Arthur D. Weston, at 7 P.M.

This meeting was the annual joint meeting with the Student Chapters of the American Society of Civil Engineers at Harvard University, Massachusetts Institute of Technology, Tufts, Brown University, Rhode Island State College and the Northeastern University Section of the Boston Society of Civil Engineers. About 175 members and guests attended, and 152 persons attended the buffet supper.

The President extended a welcome to the students and expressed appreciation of the co-operation of all those who had co-operated in obtaining so large an attendance.

The Secretary reported the election of the following members on this date:

*Grade of Member:* Clinton C. Barker, Oral J. Calderara,\* James N. De Serio,\* Frank L. Heaney,\* Waldo I. Kennerson, Oscar R. Lindgren,\* Martin J. Markham,\* Sabestino Volpe,\* Charles A. White,\* Julian H. White.\*

*Grade of Junior:* James B. Gibbs,† Richard Halloran, William H. Mitchell,† Harry Samuel Perdikis,† Walter A. Randazzo, John F. Wiseman.†

*Grade of Student:* Chester L. Harris.

The President introduced Mr. John Elliott, who outlined briefly the campaign for associate membership in the Engineering Societies of New England.

The President then introduced the speaker of the evening, Dr. Miller McClintock, Director of Bureau for Street Traffic Research, Harvard University. He gave an extremely interesting talk on "New Types of Traffic Facilities." Descriptions were given of some of the modern super highways for the rapid and expeditious handling of large volumes of motor traffic, and the fundamental principles of design to prevent interruption to traffic flow and for providing safety as well as unimpeded movement. A question period followed the talk.

The President called upon Mr. Edgar F. Copell, Traffic Engineer, Massachusetts Department of Public Works, for discussion.

The meeting adjourned at 9.20 P.M.

EVERETT N. HUTCHINS, *Secretary*.

DECEMBER 15, 1937. — A regular meeting of the Boston Society of Civil Engineers was held this evening at the Engineers Club, and was called to order at 7 P.M. by the President, Arthur D. Weston. Fifty members and guests were present. Forty persons attended the buffet supper.

The President announced the death of the following members: Neal J. Holland,

\* Transfer from Grade of Junior.

† Transfer from Grade of Student.



who died December 2, 1937, and had been a member since May 17, 1911; James A. McKenna, who died June 15, 1937, and had been a member since December 20, 1899; Edgar P. Sellw, who died October 20, 1937, and had been a member since October 17, 1888.

The Secretary reported that the following had been elected to membership on December 15, 1937:

*Grade of Member:* John H. Bowie.\*

*Grade of Student:* Arnold B. James, Grant W. Joslin, Gordon A. Thomson.

Upon recommendation of the Board of Government it was—

*Voted,* That the Board of Government be authorized to use as much of the current income of the Permanent Fund as is necessary to meet the current expenses of the Society. The President stated that this matter will be presented at the January 26, 1938, meeting for final action.

The President then introduced the speaker of the evening, Mr. Lawrence Orton, Secretary and General Director, Regional Plan Association, Inc., New York City, who gave a very interesting talk on "Large Scale Planning," with particular reference to the developments in New York City, and he outlined the extensive powers and duties of the new planning commission for the city.

The meeting adjourned at 9 P.M.

EVERETT N. HUTCHINS, *Secretary*.

### Sanitary Section

NOVEMBER 12, 1937. — A special meeting of the Sanitary Section was held this evening at the Massachusetts Institute of Technology, Cambridge. Preceding the meeting nineteen members gathered in the North Hall of the Walker Memorial for supper.

The meeting was called to order by the Chairman, Richard S. Holmgren, at 7.15 P.M. in the Eastman Lecture Hall, with thirty-eight members and guests present. Mr. A. B. Morrill, Associate Civil and Sanitary Engineer, city of Detroit, gave a very interesting talk on "Some Features of the New 420 M. G. D.

Detroit Sewage Treatment Plant." The construction of this plant is now nearly completed, and the description of the various problems encountered during construction held the interest of all who attended. Following the discussion the meeting adjourned at 8.25 P.M.

RALPH M. SOULE, *Clerk*.

### Designers Section

NOVEMBER 23, 1937. — The November meeting of the Designers Section was held jointly with the Highway Section at the Society Rooms.

The speaker was Mr. Donald W. Taylor, Research Associate in Soil Mechanics at the Massachusetts Institute of Technology. He spoke on the "Stability of Earth Slopes," discussing in detail the paper which appeared under this title in the July, 1937, JOURNAL of the Boston Society of Civil Engineers.

The talk was illustrated by lantern slides.

A question and discussion period followed the talk, and the meeting adjourned at 7.55 P.M.

The attendance was forty.

J. D. MITSCH, *Clerk*.

DECEMBER 8, 1937. — The December meeting of the Designers Section was held at the Massachusetts Institute of Technology on December 8, 1937, at 7 P.M.

The meeting consisted of a talk by Prof. John B. Wilbur, Associate Professor of Civil Engineering at Massachusetts Institute of Technology, and an inspection of the new structural model laboratory. In the talk Professor Wilbur discussed the instruments and materials used in the testing and building of structural models.

The inspection of the various problems being carried on in the laboratory proved interesting and instructive, and the meeting closed at 8.30 P.M.

The attendance was forty-five.

J. D. MITSCH, *Clerk*.

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\* Transfer from Grade of Junior.

## APPLICATIONS FOR MEMBERSHIP

[January 10, 1938]

THE By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

### *For Admission*

BECKER, ROBERT MAXWELL, Brighton, Mass. (Age 24, b. Boston, Mass.) Graduated, 1934, from Massachusetts Institute of Technology with S.B. degree in building engineering and construction; June, 1934, to April, 1935, laboratory assistant to Prof. W. C. Voss at Massachusetts Institute of Technology, on masonry materials research; May, 1935, to July, 1935, engineer and quantity clerk with Aberthaw Company, on Suffolk Downs grandstand; September, 1935, to March, 1936, architectural draftsman and specification writer with D. J. Abrahams, Boston, and assistant to T. F. McSweeney, consulting engineer, Boston, part time with each. March, 1936, to June, 1936, in charge of field engineering with Temple & Crane, Inc., first-class factory building, North Smithfield, R. I.; June, 1936, to January, 1937, superintendent and estimator with Arnold Hartmann, Newton Center (Oak Hill Village), building eight

large residences. January, 1937, to date, with H. E. Cline Construction Company, Boston, as engineer and estimator, in charge of new apartment building, 1960-1980 Commonwealth Avenue, Brighton, and superintendent of construction of addition to W. T. Grant Building, Newport, R. I. Refers to *J. B. Babcock, J. G. Dietz, W. M. Fife, J. D. Mitsch, W. C. Voss.*

DAVIS, ERNEST LEWIS FREDERICK, Boston, Mass. (Age 55, b. Boston, Mass.) Graduated from Mechanics Arts High School, Boston, in 1902; then employed as draftsman in structural iron shop preparing detail plans for shop and erection use; April, 1904, to September, 1911, with Metropolitan Park Commission as rodman, transitman and resident engineer; September, 1911, to November, 1911, with Massachusetts Highway Commission as resident engineer; November, 1911, to May, 1913, with Massachusetts State Board of Health in charge of party on Neponset River improvement; May, 1913, to February, 1921, with Metropolitan Park Commission, as chief of party and resident engineer in charge of surveys and construction of Cottage Farm bridge and temporary Neponset River bridge; February, 1921, to the present time, first, in charge of drafting room, then office assistant to chief engineer, Massachusetts Department of Public Works; now associate civil engineer. Refers to *A. W. Dean, G. H. Delano, R. K. Hale, E. N. Hutchins.*

MAYER, DAVID, Cambridge, Mass. (Age 22, b. New York, N. Y.) Graduated, 1936, with degree of A.B., from Harvard College, and Harvard School of Engineering, with degree of M.S., in 1937; during summers of 1935 and 1936, with Bethlehem Steel Company, Fabricating Division, Pottstown, Pa., apprentice in drafting room; now engaged in building materials research, as research assistant at Massachusetts Institute of Technology. Refers to *Arthur Casagrande, G. M. Fair, A. Hartlein, H. M. Westergaard.*

RICH, EDWIN STAFFORD, Norwood, Mass. (Age 30, b. Boston, Mass.) 1926, graduated from Wentworth Institute, course in architectural construction; 1928, completed Lowell Institute course in

buildings; 1926-28, structural detailer with New England Structural Company; 1928-31, structural detailer and estimator with Palmer Steel Company, Springfield, Mass.; 1931-32, estimator with A. O. Wilson Structural Company; 1932-33, sales engineer with Frigidaire Sales Corporation; 1933-34, timekeeper with West Construction Company; 1934-35, timekeeper and job engineer with Blakeslee, Rollins Corporation; 1935-37, New England District Construction Supervisor (Safety Engineering Department) of Liberty Mutual Insurance Company; 1937 to date, engineer with Blakeslee, Rollins Corporation. Refers to *M. F. Brown, C. A. Farwell, J. W. Howard, S. Huckins.*

*For Transfer from Grade of Junior*

CAVAZZONI, JOSEPH FRANCIS, Somerville, Mass. (Age 29, b. Bologna, Italy.) Graduated from Somerville, Mass., High School, 1926, and Northeastern University, 1930, with degree of B.C.E.; May, 1930, to July, 1934, with John Williams, contractor, as estimator and superintendent in various types of construction, including sewers, roads, drains and water mains in cities in Massachusetts; July, 1934, to January, 1937, with A. Singarella, contractor, as estimator on building sewers, water works, concrete and steel bridges, culverts. Since January, 1937, in private practice, making plans for various structures and surveying. Refers to *H. B. Alword, S. O. Baird, Jr., J. J. Casey, A. E. Everett, Jr.*

*For Transfer from Grade of Student*

PERRY, LESTER S., Boston, Mass. (Age 24, b. Somerville, Mass.) Graduated from Northeastern University, 1937, with degree of B.S. in civil engineering. July to November, 1936, as co-operative student with Department of Interior National Park Service as a special enrollee in the Civilian Conservation Corps as rodman, transitman and draftsman; February to April, 1937, with W. W. Churchill, surveyor, Milton, Mass., as transitman and chief of party; September, 1937, to present

time, draftsman with Water Bureau; Metropolitan District, Hartford, Conn. Refers to *H. B. Alword, C. O. Baird, Jr., E. A. Everett, Jr., E. A. Gramstorff, H. A. Phillips.*

## ADDITIONS

### *Members*

CLINTON C. BARKER, 33 Crossman Avenue, Beach Bluff, Mass.  
JOHN H. BOWIE, 36 Lantern Lane, Milton, Mass.  
ORALL J. CALDERARA, Box A, Templeton, Mass.  
GEORGE A. MACDONALD, 930 Summit Avenue, River Edge, N. J.  
MARTIN J. MARKHAM, 35 Lincoln Street, Stoneham, Mass.  
CHARLES A. WHITE, North Falmouth, Mass.  
JULIAN H. WHITE, 84½ Washington Street, Brewer, Maine.

### *Juniors*

RICHARD HALLORAN, 95 Dedham Street, Newton Highlands, Mass.  
HARRY S. PERDIKIS, 419 Federal Building, Providence, R. I.  
JOHN F. WISEMAN, 26 Clayton Street, Malden, Mass.

### *Students*

CHESTER L. HARRIS, 922 Beacon Street, Boston, Mass.  
ARNOLD B. JAMES, 923 Beacon Street, Boston, Mass.  
GRANT W. JOSLIN, 33 Westminster Avenue, Arlington, Mass.  
GORDON A. THOMSON, East Gay Street, Dedham, Mass.

## DEATHS

NEAL J. HOLLAND . . . Dec. 2, 1937  
JAMES A. MCKENNA . . . June 15, 1937  
EDGAR P. SELLEW . . . Oct. 20, 1937

## BOOK REVIEWS

*"Bridges in History and Legend,"* by Wilbur J. Watson and Sara R. Watson. 248 pages. Published by J. H. Jansen, Cleveland, Ohio. Price, \$3.50.\*

This is a non-technical book designed to show the significance of bridges in civilization, in the thought of man, and in his art. There are general descriptions of historical bridges and many pen and ink sketches illustrating these structures, but such material is only incidental to the main purpose. The principal emphasis is on legends and stories about bridges, and on events which have occurred on famous bridges. It seems that every famous bridge has had a poem written about it at some time or other. Many poets have used the bridge as a symbol in writing about life. Poetry is prominent throughout the volume, and comprises about ten per cent of the total number of printed lines in the book.

The earliest legendary references are to Rainbow Bridges, which have a religious symbolism and span from this to another world. These structures sustain the good and let fall the evil. There follow stories about Devil's Bridges, of which there are many existing specimens and whose construction was made possible only through the intervention of the devil, who has claimed a soul for his part in the deal. Later, the gods of both heaven and hell are concerned, and the bridge builders are church officials who construct Sacred Bridges to span streams which are identified with agents of Satan.

From very early times bridges have been important in military operations and, therefore, War Bridges are given their proper amount of space. This part of the book is not legendary but historical, and includes incidents from the building of the pontoon bridge over the Hellespont in 480 B.C. to the murder on the bridge at Sarajevo which precipitated the World War.

An entire chapter is devoted to the Old London Bridge, which was partly financed

by a tax on wool. It is interesting to know that at one time tolls were collected from boats passing under this bridge as well as from traffic passing over it. Rentals from buildings on the bridge was another source of revenue. In medieval times a source of revenue for chapel or sacred bridges was the sale of indulgences.

In connection with Toll Bridges one learns that the Old Cambridge Bridge, built in 1786 on the site of the present Longfellow Bridge, paid principal, interest and about \$7,000 surplus to each original share in forty years. This is a contrast to present-day toll bridges, which do not enjoy that kind of prosperity.

A chapter on Peace Bridges contains many stories of human interest. Medieval bridges were built for peaceful purposes, but had to be strongly fortified. The Karlsbrücke at Prague took one hundred and forty-six years to build. A bridge is involved in the origin of the card game "bridge whist" according to one story.

One chapter contains brief sketches of famous Bridge Builders. Another traces the development of bridge construction in the last century.

It is gratifying for an engineer to learn that bridges have participated to such an important degree in the history of civilization and that they have been found fit subjects by so many poets, artists and writers. While not technical, this book can be of value to an engineer in developing background.

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*"A Decade of Bridges,"* by Wilbur J. Watson. 125 pages. Published by J. H. Jansen, Cleveland, Ohio. Price, \$4.50.\*

In the design of bridges it is helpful to know what has been done previously in the projection of similar structures, what difficulties were encountered, and how they were met. This book provides answers to such questions by presenting a photographic record of the progress of bridge building in the decade 1926-1936, together with general descriptions and comments on engineering and architectural

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\* Reviewed by Eugene Mirabelli, Assistant Professor of Structural Design, Massachusetts Institute of Technology, Cambridge, Mass.



aspects of some one hundred notable bridges.

The illustrations are of both European and American structures, but only those are included which contain novel engineering or architectural features or serve to illustrate modern tendencies in bridge design. Reference is made to the more interesting points met in the design, financing, construction and operation of these bridges. Physical data appears throughout the volume, and occasionally there is a historical reference. Designers, consultants and contracting firms connected with each project are noted.

For more detailed information about the bridges included in the text the reader is referred to the appendix, which contains a selected index to descriptions in technical journals.

The principal feature of the book is probably its collection of excellent photographs. This volume, together with the author's previous volume on "Bridge Architecture," provides a continuous photographic history of the world's famous bridges from the earliest times to the year 1936.

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*"Theory of Statically Indeterminate Structures," by Walter Maxwell Fife and John Benson Wilbur, Associate Professors of Civil Engineering at Massachusetts Institute of Technology. 245 pages. Published by McGraw-Hill Book Company, New York, 1937. Price, \$3.50.\**

In their new text, "Theory of Statically Indeterminate Structures," Professors Fife and Wilbur of the Massachusetts Institute of Technology have for the first time made accessible to the average American engineer the rigorous structural thinking of the great German analyst, Dr. Heinrich Müller-Breslau. This is a mature, closely reasoned treatise which the beginner in the indeterminate field will find somewhat more difficult than those texts which approach the subject in the freer fashion usual in this country, — a difficulty which should be compensated for by increased competency and thoroughness of understanding. The graduate student, with the

usual undergraduate preparation in indeterminate analysis, will traverse these pages with relative ease and pleasure, finding that they open up new relations and points of view which integrate "in a thorough and cohesive manner the principles which underlie the analysis of statically indeterminate structures." Throughout the book there is a generous use of numerical examples which give concreteness and clarity to the discussions.

The first chapter deals with the usual basic concepts and theorems presented in a very unusual and thorough manner. The general case of a three-dimensional frame with joints capable of resisting bending is considered, with six independent elements of stress at any section and six independent components of deflection for the end of any member. The unknowns accordingly number six times the sum of the joints and bars. For each joint six equations of equilibrium may be written, and for each member six geometric equations relating the conditions of end deflection. This manner of setting up basic relations is probably new to most American readers. The conditions for status as statically determinate, stable, unstable, statically indeterminate, are carefully investigated in detail from this general basis. The study of stability criteria is of especial interest, since their need was brought to our attention by Mr. Alfred Waidelich soon after the appearance of the Wickert truss.

The law of virtual work and those of Clapeyron and Betti are derived also with unusual thoroughness and generality. Maxwell's law of reciprocal deflections is stated briefly as a special case under Betti's law.

The second chapter is headed "Deflections," and considers the first portion of the general problem of analysis, the determination of the shape of the distorted loaded structure. The computation methods given comprise those by virtual work and by Castigliano's law, the Williot-Mohr graphical method, and Müller-Breslau's bar chain method. The basic theorems of moment-area and elastic weights are developed, having regard to shear as well as to bending strain.

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\* Reviewed by Hale Sutherland, Professor of Civil Engineering, Lehigh University.

The second half of the general problem of structural analysis, the determination of stress at any point in a structure, is considered in Chapter III, and the discussion probably conforms more closely to the usual methods of presentation than do the preceding chapters. Since only sixty-nine pages are given to the stress analysis of trusses and rigid frames, it is evident that only basic methods and the more common situations are discussed, omitting such topics as members of variable section, multiple story frames under lateral loading, and frames of multiple span, to mention three "headaches" of not uncommon appearance in structural offices.

The first method of analysis considered utilizes the familiar equations of consistent distortion given by application of the law of virtual work. When considering frames with moment-resisting joints shear distortion is included. The use of the elastic center is studied in connection with single-span, one-story frames, and it is demonstrated that this leads to the method of column analogy of Prof. Hardy Cross. A very fine development is given for the application of Castigliano's law to indeterminate trusses and frames, which, for the usual situation of unyielding supports, results in that essential basic — the familiar method of least work.

The standard equations for end moments for members with restrained ends are developed with exceptional clarity as to significance, and from these equations the methods of slope-deflection and the three-moment equation are derived. The last twelve pages of the chapter are given to Professor Cross' method of moment distribution as applied to continuous beams and to one and two story frames.

The fourth chapter (thirty-eight) pages is given to the practically important topic of influence lines for statically indeterminate structures. The possibilities of labor reduction by systematization of work are illustrated by the computation of an influence line by successive positions of the unit load. The use of the elastic curve and Maxwell's law in constructing influence lines for deflection, bar stresses and reactions is here illustrated for trussed

structures, both singly and doubly indeterminate. The discussion of influence lines for fixed ended, restrained and continuous beams leads to a brief exposition of methods of mechanical analysis. The chapter concludes with a brief statement of the fixed-point method for continuous beams.

The fifth and concluding chapter deals with secondary stresses in trusses, giving the variation of the Manderla method suggested by Winkler, the Mohr semi-graphical method, and Prof. Hardy Cross' method of moment distribution.

In their preface the authors point to the criticism currently leveled at the so-called classic methods of analysis, and state their conviction that the principles underlying these methods furnish "the best possible foundation for a knowledge of structural theory, and that familiarity with them is essential to an understanding of structural behavior." They do not decry nor neglect the great contributions of Professor Cross, but their major interest and effort is given to the classic methods, and they have accomplished a very fine piece of work in support of their thesis. It may be questioned, however, whether the situation is quite as this preface statement implies. In his method of moment distribution, Professor Cross introduced no new principles, but rather presented a revolutionary procedure which immensely facilitates both the application of basic principles and the visualization of structural action. Our authors emphasize the first point but not the second. The structural analyst should recognize that he deals with a compact body of basic principles involving several differing methods of mathematical application, whose relative values are to be judged on the basis of laboriousness and the degree of obscuration of the physical action of the loaded structure involved in the processes of computation. It is probable that this point of view would have led in no way to any change or curtailment of the treatment the authors have given the "classic" in this text, but rather to a more extensive and thorough study of moment distribution which by reason of its basic nature should henceforth rate as a truly *classic* method.



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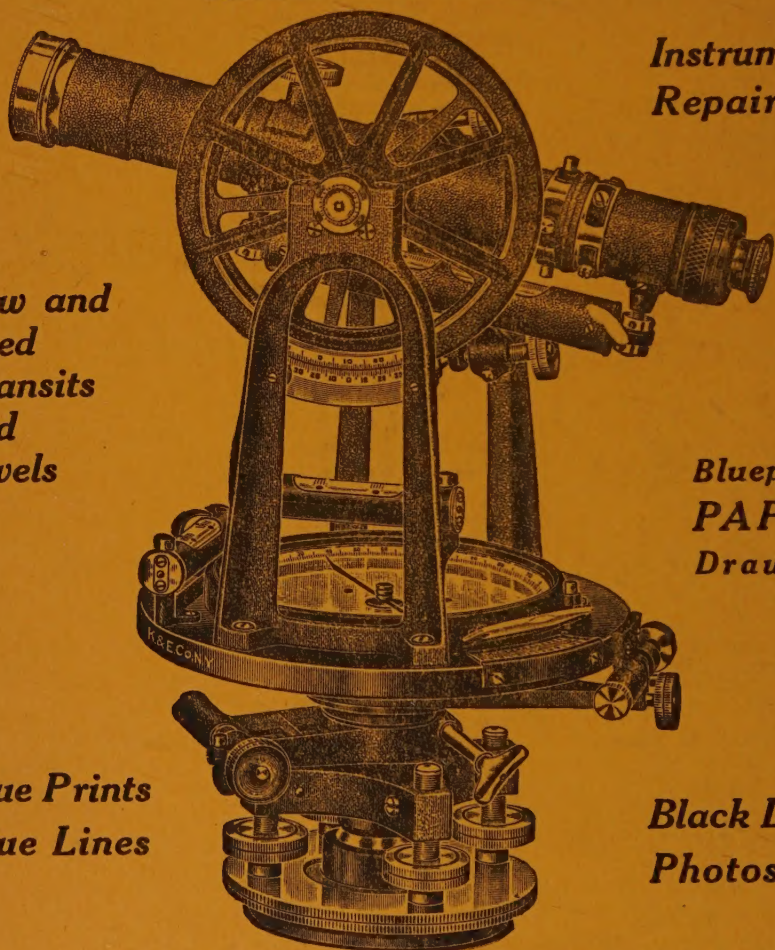
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